Interface Shear Resistance of Clustered Shear Connectors for Precast Concrete Bridge Deck Systems

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Abstract

The use of full-depth precast concrete deck systems in bridge construction has been increasing in recent years due to their high production quality, reduced construction duration and its impact on the traveling public, possible weight reduction, and lower life-cycle cost. Precast concrete deck systems can be either composite or non-composite with the supporting steel/concrete girders. Composite systems are more common due to their superior structural performance and reduced overall superstructure depth and cost. Most of the composite systems require the use of clustered shear connectors to reduce the number of field-cast connections and simplify panel production and erection. The current prediction models of interface shear resistance in most bridge design codes were developed for continuous shear connectors in cast-in-place bridge deck systems. There is a need to evaluate the accuracy of these models when applied to predict the interface shear resistance of clustered shear connectors. In this study, the results of 162 push-off experiments conducted in North America, Europe, and South Korea were used to compare the interface shear resistance prediction models provided by AASHTO LRFD, *fib* MC, Eurocode-2, and CSA-S6 bridge design codes. Comparisons indicated that all design codes provide conservative estimates for interface shear resistance of clustered shear connections when compared to the measured data.

Parameters affecting the interface shear resistance of clustered shear connectors were also identified.

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Introduction

Interface shear transfer between concrete bridge decks and steel/concrete bridge girders in composite systems has been heavily investigated in the last 50 years (Anderson, 1960; Hanson, 1960; Birkeland and Birkeland, 1966; Hoefbeck et al., 1969; Mattock and Hawkins, 1972; Paulay et al., 1974; Mattock et al., 1976; Walraven et al., 1987; Loov and Patnaik, 1994; Mattock, 2001; Khan and Mitchell, 2002; and Khan and Slapkus, 2004). Current design code provisions are based on the outcomes of these investigations, which were conducted using continuous shear connectors (i.e. studs or bars) along the interface between the cast-in-place concrete deck and steel/concrete girders. However, little-to-no research was done to evaluate the applicability of these code provisions to clustered shear connectors commonly used in precast concrete deck systems. Therefore, the objective of this paper is to evaluate the applicability of existing interface shear code provisions to predict the capacity of clustered shear connectors. A database of 162 push-off test results is used to examine the predictability of interface shear resistance of clustered shear connectors using four international code provisions: AASHTO LRFD (2014); *fib* MC 2010; Eurocode-2 (2004); and CSA-S6-06. Also, the effect of key parameters, such as concrete compressive strength, reinforcement ratio, and yield strength, on the interface shear resistance are studied.

Background of Interface Shear

Birkeland and Birkeland (1966) were the first to propose a linear expression to evaluate the ultimate interface shear stress of concrete interfaces. Figure 1 shows the shear friction model proposed by Birkeland and Birkeland (1966), which can be presented by following expression:

$$v_u = \rho f_y \tan \varphi = \rho f_y \mu \tag{1}$$

where, v_u is the interface shear resistance; ρ is the reinforcement ratio; f_y is the yield strength of the reinforcement; and φ is the internal friction angle. The tangent of the internal friction angle is also known as coefficient of friction, and the term ρf_y is known as clamping stresses. This expression was proposed for smooth concrete surfaces, artificially roughened concrete surfaces, and concrete-to-steel interfaces. The coefficient of friction was empirically determined from experimental testing results, varying with the surface preparation, and it was defined for several situations, namely: (a) μ =1.7, for monolithic concrete; (b) μ =1.4, for artificially roughened construction joints; and (c) μ =0.8–1.0, for ordinary construction joints and for concrete to steel interfaces.

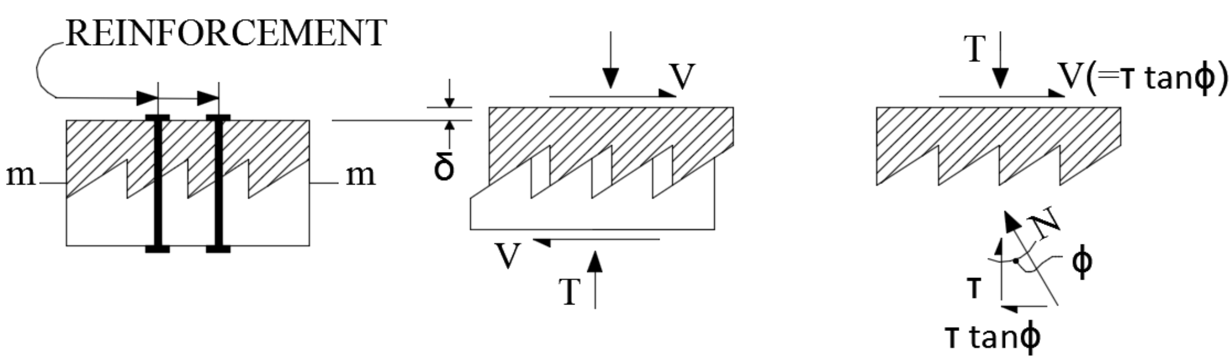


Figure 1: Shear friction model (Birkeland and Birkeland, 1966)

This expression, when first proposed by Birkeland, was limited to the following conditions: $f_y \leq 60\ ksi,\ \rho \leq 1.5\%,\ v_u \leq 800\ \ and\ \ f_c \geq 4000\ \ (psi)$. In Figure 1, as the slip progresses, a normal displacement (δ) occurs and this displacement can be large enough to cause yielding of the shear connector in tension. Different design codes have adopted this equation with minor modifications to the shear friction coefficients along with considering concrete cohesion contribution to the interface shear resistance.

Randi (1997) developed the extended shear friction model, which is considered a significant contribution to the accuracy of interface shear design expressions (*fib* MC 2010). This design expression accounts for contribution of concrete cohesion/aggregate interlock, shear friction, and dowel action of the shear reinforcement. *fib* MC 2010 for concrete structures adopted this method for calculating the interface shear resistance of two concretes cast at different times. Figure 2 shows a schematic representation of the interface shear resisting mechanisms.

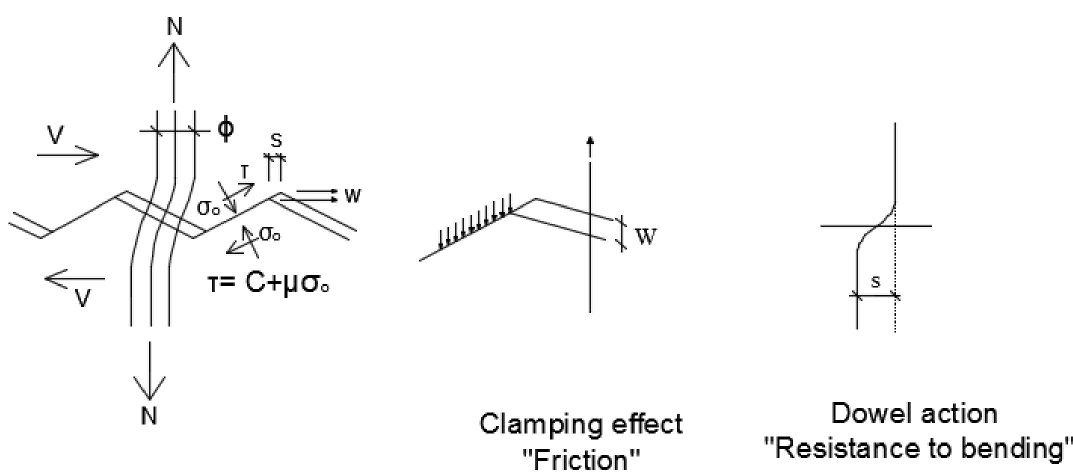


Figure 2: Interface shear resistance mechanism (Randi, 1997)

The proposed design expression is as follows:

$$\tau_{Rdi} = c_r f_{ck}^{1/3} + \mu\sigma_n + k_1\rho f_{yd}(\mu \sin\alpha + \cos\alpha) + k_2\rho\sqrt{f_{yd}f_{cc}} \leq \beta_c v f_{cc} \quad (2)$$

$$v = 0.55(\frac{30}{f_{ck}})^{1/3} < 0.55$$

where,

τ_{Rdi} is the ultimate shear stress at the interface

ρ is ratio of reinforcement crossing the interface ($\rho = As/Ac$);

β_c is a coefficient for the strength of the compression strut (see also Table 2);

v is the effectiveness factor for the concrete;

c_r is the coefficient for aggregate interlock effects at rough interfaces (see also Table 2);

k_1 is the interaction coefficient for tensile force activated in the reinforcement or the dowels (see also Table 2);

k_2 is the interaction coefficient for flexural resistance (see also Table 2);

μ is the friction coefficient (see also Table 2);

α is the inclination of the reinforcement crossing the interface

σ_n is the (lowest expected) compressive stress resulting from an eventual normal force acting on the interface.

f_{cc} cylinder compressive strength of concrete under uniaxial stress, however, f_{cd} should be used in design (design value of f'_c), N/mm^2;

f_{ck} characteristic value of compressive strength of concrete;

f_{yd} is the design yield strength of reinforcing steel in tension.

Table 1: Definition of surface roughness (*fib* MC 2010 Table 6.3-1)

Category	***R_t, mm (in.)**
Very smooth (e.g., cast against steel formwork)	not measureable
Smooth (e.g., untreated, slightly roughened)	< 1.5 (1/16)
Rough (e.g., sand blasted, high pressure water blasted etc.)	≥ 1.5 (1/16)
Very rough (e.g., high pressure water jetting, indented)	≥ 3 (1/8)

*R_t is the "peak-to-mean" surface roughness

Table 2: Coefficients for different categories of surface roughness (*fib* MC 2010 Table 7.3-2)

Surface Roughness Category	c_r	k_1	k_2	β_c	μ	
					f_{ck} ≥ 20 Mpa (2.9 ksi)	**f_{ck} ≥ 35 Mpa (5 ksi)**
Very rough	0.2	0.5	0.9	0.5	0.8	1
Rough	0.1	0.5	0.9	0.5	0.7	
Smooth	0	0.5	1.1	0.4	0.6	
Very smooth	0	0	1.5	0.3	0.5	

The first term, cohesion, is related to the contribution of interlocking between aggregates. The second and third terms, friction, is related to the contribution of the horizontal relative slippage between concrete parts and is influenced by the surface roughness and the normal stress due to axial elongation of shear reinforcement at the shear interface. The forth term, dowel action, is related to the contribution of flexural resistance of the shear reinforcement crossing the interface due to bending and horizontal deformations of the reinforcement.

Code Provisions

Four international bridge design codes are considered for evaluating the interface shear resistance of clustered shear connections. These codes are AASHTO LRFD (2014); *fib* MC 2010; Eurocode 4; and CSA-S6-06. The equations used in each code as well as a short description of the governing parameters are presented in the following sections. It is worth noting that AASHTO LRFD (2014), Eurocode 4, and CSA-S6-06 code provisions are based on the shear friction model developed by Birkeland and Birkeland (1966), while *fib* MC 2010 code provisions are based on the extended shear friction theory developed by Randi (1997).

AASHTO LRFD (2014)

AASHTO LRFD bridge design specifications (Article 5.8.4) provide equations to calculate the nominal shear resistance across a given plane at: an existing or potential crack; an interface between dissimilar materials; an interface between two concretes cast at different times; or the interface between different elements of the cross-section. AASHTO LRFD uses a modified shear-friction model accounting for the contribution of cohesion and/or aggregate interlock, given by the first term of the equation. The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c) \tag{3}$$

The nominal shear resistance, V_{ni}, shall not be greater than the lesser of:

$$K_1 f_c' A_{cv} \text{ or } K_2 A_{cv}$$

where,

V_{ni} = nominal shear resistance, lb

c = cohesion factor (see Table 3)

A_{cv} = area of concrete considered to be engaged in interface shear transfer ($b_{vi}.L_{vi}$), in.^{2}

μ = friction factor (see Table 3)

A_v = area of reinforcement crossing the shear plane within the area A_{cv}, in^2

f_y = yield stress of transverse reinforcement, psi

P_c = permanent net compressive force normal to the shear plane, lb

b_{vi} = interface width considered to be engaged in shear transfer, in.

L_{vi} = interface length considered to be engaged in shear transfer, in.

K_1 = fraction of concrete strength available to resist interface shear, (see Table 3)

K_2 = limiting interface shear resistance specified in Table 3

Table 3: Coefficients for different interface types (AASHTO LRFD, 2014)

Interface type	***c* (ksi)**	**μ**	**K_1**	**K_2 (ksi)**
Monolithic concrete	0.40	1.4	0.25	1.5
CIP concrete slab on clean intentionally roughened concrete girder surfaces, R = 0.25 in.	0.28	1.0	0.3	1.8
Concrete placed against clean concrete surfaces, R = 0.25 in.	0.24	1.0	0.25	1.5
Concrete placed against clean concrete surfaces, R = 0.0 in.	0.075	0.6	0.2	0.8
Concrete placed against as-rolled structural steel and free of paint, anchored by headed studs or reinforcement bars.	0.025	0.7	0.2	0.8

CIP = cast-in-place; R = roughness amplitude.

fib **Model Code 2010**

fib model code for concrete structures 2010 (*fib* MC 2010) provides basic concrete-to-concrete load transfer across interfaces in Section 6.3 with the corresponding design rules and parameters in Section 7.3.3.6. Different potential failure mechanisms contributing to the interface shear resistance were considered such as adhesive bond, aggregate interlock, friction, and dowel action. It is worth noting that this code is considered the first to include the dowel action contribution to the interface shear resistance. Refer to the background of interface shear Section of this paper for *fib* MC 2010 interface shear equation and parameters (Figure 2 and Equation 2).

Eurocode 2 (EN 1992-1-1:2004)

When a combination of precast elements and in-situ concrete is used, the resistance to longitudinal shear should also be determined in accordance with EN 1992-1-1: 2004, section 6.2.5 to check the shear strength at the interface between concrete cast at different times, which is given by the following equation:

$$V_{Rdi} = cf_{ctd} + \mu\sigma_n + \rho f_{yd}(\mu \sin\alpha \cos\alpha) \leq 0.5vf_{cd} \tag{5}$$

$$v = 0.6\left[1 - \frac{f_{ck}}{250}\right]\ (f_{ck}\ in\ MPa)\ Or = 0.6\left|1 - \frac{f_{ck}}{36.26}\right|(f_{ck}\ in\ ksi) \tag{6}$$

where,

V_{Rdi} is the design shear resistance at the interface

c and μ are factors which depend on the roughness of the interface (see Table 4)

f_{ctd} is the design tensile strength

f_{ck} is the characteristic compressive cylinder strength of concrete at 28 days

f_{yd} is the design yield strength of reinforcement

f_{cd} is the design value of concrete compressive strength

$\rho = A_s/A_i$

A_s is the area of reinforcement crossing the interface, including ordinary shear reinforcement (if any), with adequate anchorage at both sides of the interface.

A_i is the area of the joint (area of concrete across the interface)

α is the angle of interface shear reinforcement measured from the horizontal interface shear plane

σ_n is the stress per unit area caused by the minimum external normal force across the interface that can act simultaneously with the shear force, positive for compression, such that $\sigma_n < 0.6\, f_{cd}$, and negative for tension. When σ_n is tensile, f_{ctd} should be taken as 0.

Section 6.6.6.1 (EN 1994-2:2005) specifies that longitudinal shear failure and splitting of the concrete slab due to concentrated forces applied by the connectors shall be prevented in order to achieve the interface shear resistance predicted by the previous equation.

Table 4: Coefficients for different surface roughness (EC-4)

Surface Roughness	***c***	***μ***
Very smooth*	0.025 to 0.10	0.5
Smooth**	0.20	0.6
Rough***	0.40	0.7

*a surface cast against steel, plastic or specially prepared wooden molds

**a slip formed or extruded surface, or a free surface left without further treatment after vibration

*** a surface with at least 3 mm (1/8 in.) roughness at about 40 mm (1.6 in.) spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behavior

CSA-S6-06

CSA-S6-06 (Canadian Highway Bridge Design Code) clause 8.9.5.1 specifies that a crack shall be assumed to occur along the shear plane and the relative displacement shall be considered to be resisted by cohesion and friction maintained by the shear-friction reinforcement crossing the crack. The shear resistance of a plane, v, may be calculated as:

$$v = \varphi_c (c + \mu\sigma) \leq 0.25\varphi_c f_c' \text{ or } 6.5 \text{ MPa (940 psi)} \quad (7)$$

where,

φ_c is the resistance factor for concrete

σ is the compressive stress across a shear-friction plane, MPa

μ is the friction coefficient (see Table 5)

c is the cohesion strength, MPa (see Table 5)

The value of σ in Clause 8.9.5.1 shall be calculated as follows:

$$\sigma = \rho_v f_y + \frac{N}{A_{cv}} \quad (8)$$

where,

ρ_v is the ratio A_{vf}/A_{cv}

A_{cv} is the area of concrete resisting shear transfer

A_{vf} is the area of shear-friction reinforcement

f_y is the specified yield strength of interface shear reinforcement

N is the unfactored permanent load normal to the interface area (taken as positive for compression and negative for tension)

Table 5: Coefficients for different interface type (CSA-S6-06)

Interface type	**C, MPa (psi)**	$\boldsymbol{\mu}$
Concrete placed against hardened concrete with clean surface, but not intentionally roughened	0.25 (36)	0.6
Concrete placed against hardened concrete with clean surface and intentionally roughened to a full amplitude of about 5 mm (13/64 in.) and a spacing of about 15 mm (5/8 in.)	0.50 (72)	1.0
Concrete placed monolithically	1.00 (145)	1.4

Effect of design parameters on interface shear resistance

Interface shear resistance prediction models usually include the following parameters: specified concrete compressive strength (f'_c), tensile yield strength of interface shear reinforcement (f_y), and ratio of interface reinforcement (ρ). The effect of these three parameters on the interface shear resistance was studied using a database of 162 push-off tests conducted by the authors (Tawadrous, 2017) and others obtained from the literature. In all these tests, the girder component of the specimen was fixed, while the deck component was pushed off parallel to the interface between the two components. It should be noted that the effect of interface type/surface preparation was not considered in this study as the location of critical interface shear section was taken as shown in Figure 3, which is always a monolithic concrete interface. The critical section location was determined based on authors' observations and reports by others in the literature regarding the most common failure plane in push-off tests of pocketed connections.

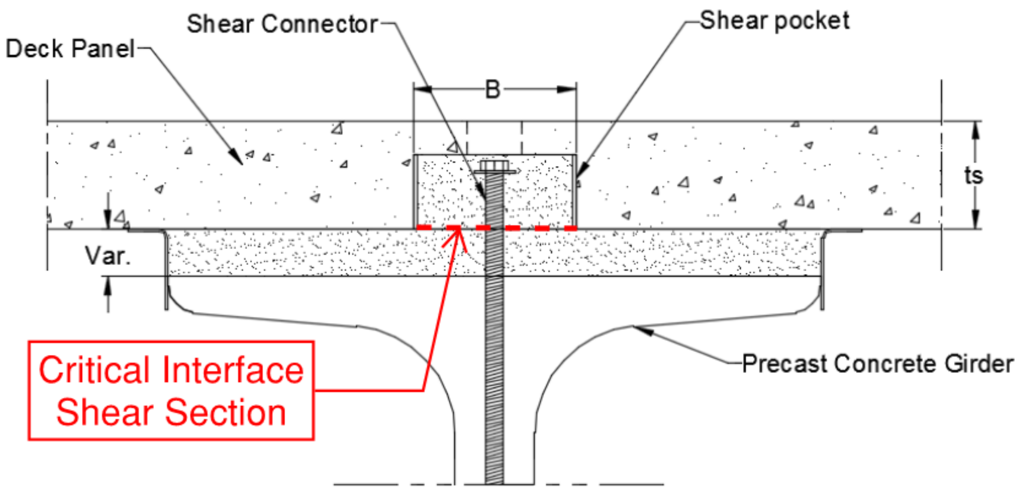


Figure 3: Location of the critical interface shear section in pocketed connections

In order to study the effect of concrete compressive strength (f_c) on the interface shear resistance of clustered shear connectors, the 162 data points were plotted in Figure 4, where shear stress (v) is on y-axis and (f_c) is on x-axis. The data are widely scattered and the general trend shows that concrete compressive strength no correlation with the interface shear resistance. This justifies the absence of concrete compressive strength from the interface shear resistance equations in most design codes. Concrete compressive strength is usually considered in defining the upper limit on the interface shear resistance. To further investigate the effect of concrete compressive strength, the same data were plotted in Figure 5 but for three categories of ρ: I) less than 2%; II) between 2 and 4%; and III) higher than 4%. Figure 5 shows that concrete compressive strength has no correlation with the interface shear resistance for categories I and II. However, concrete compressive strength has slightly higher correlation ($R^2 = 0.22$) with the interface shear resistance when reinforcement ratio exceeds 4%. This could be attributed to the high tri-axial compressive stresses exerted by large shear connector on the concrete through bearing, which are dependent on the concrete compressive strength.

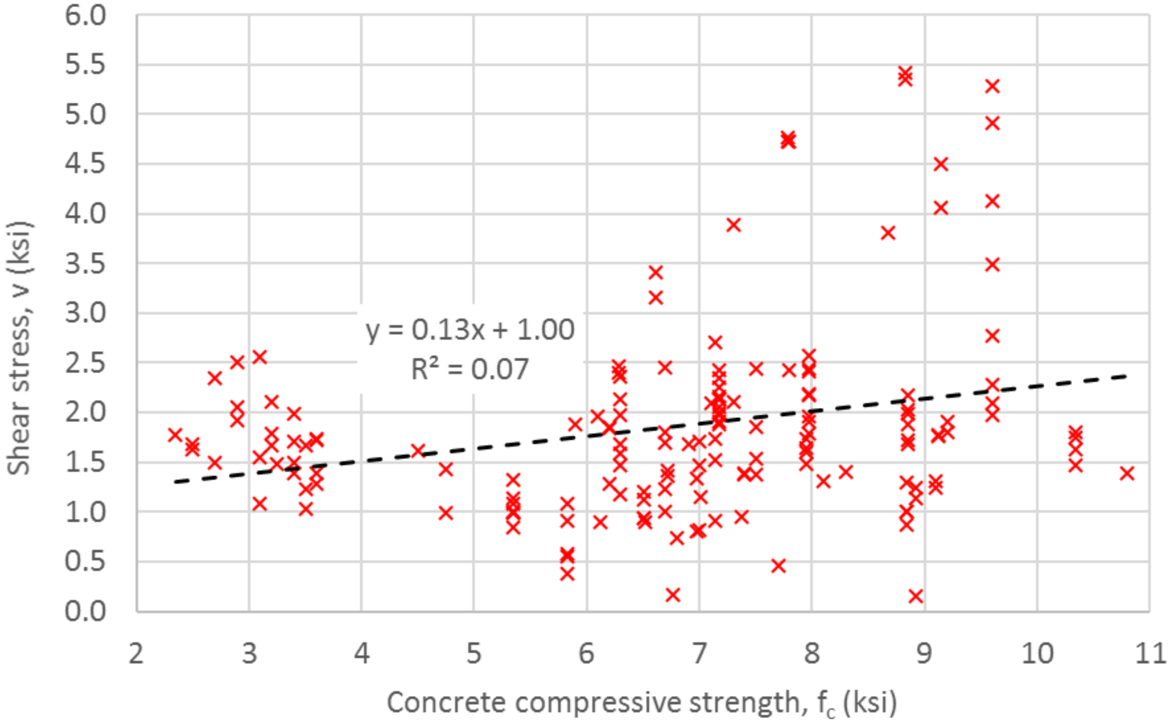


Figure 4: Interface shear stress versus concrete compressive strength (f_c is concrete compressive strength at testing time)

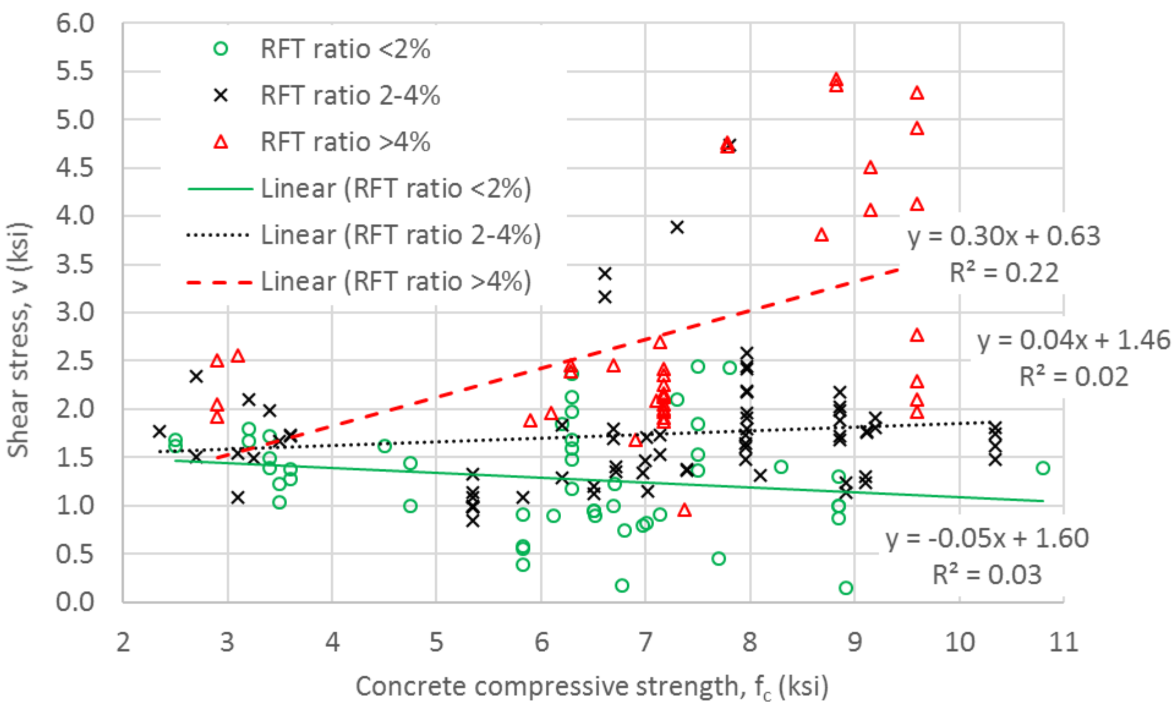


Figure 5: Interface shear stress versus concrete compressive strength for different reinforcement (RFT) ratios (f_c is concrete compressive strength at testing time)

The same set of data was plotted to study the effect of reinforcement ratio on the interface shear resistance as shown in Figure 6. Interface shear stress was plotted on the y-axis and the percentage of interface shear reinforcement area over the interface area at the critical section was plotted on the x-axis. The data was grouped by the characteristic yield strength of the shear connectors. This figure shows that interface shear stress increases as the interface shear reinforcement ratio increases. However, the yield strength of the shear connectors does not seem to have a significant effect on the interface shear resistance, which agrees with other researchers (Harries et al., 2012) and design codes. For example, AASHTO LRFD limits the maximum tensile yield strength of the shear connectors to 60 ksi. In addition, other codes such as the European and Canadian codes limit stress on concrete interface, which minimizes the advantage of using shear connectors with high yield strength.

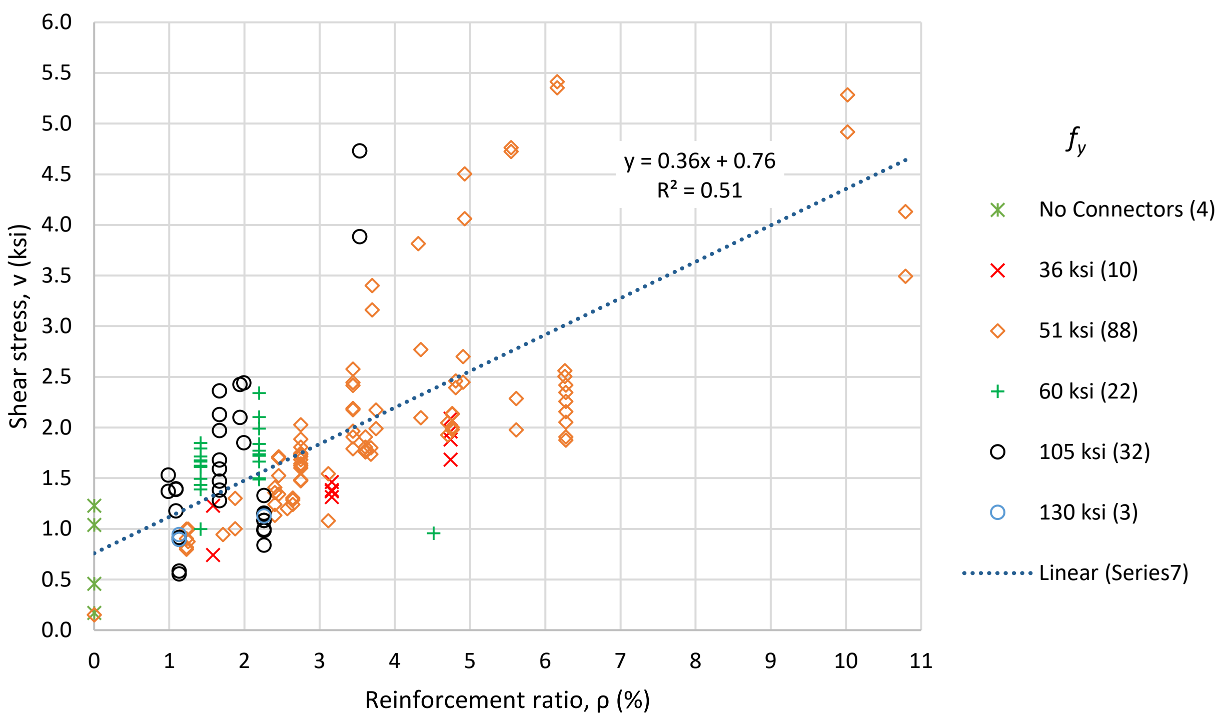


Figure 6: Interface shear stress versus reinforcement ratio

Code Comparisons

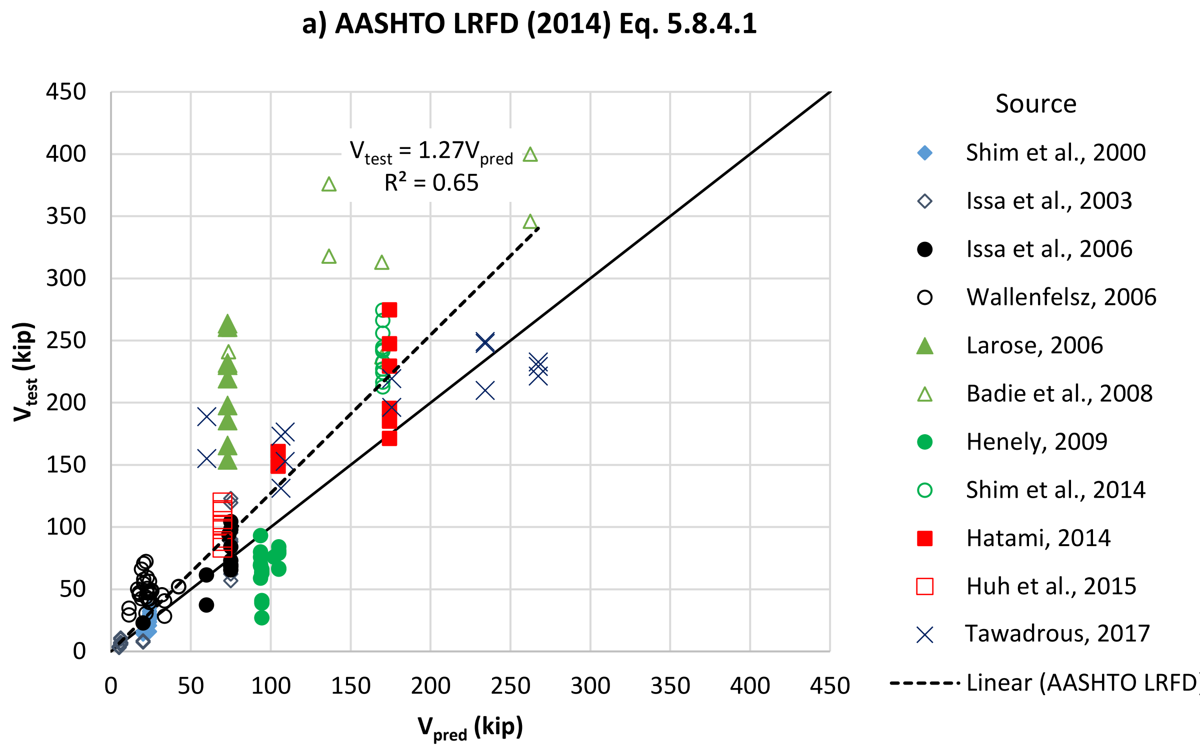
Four international bridge design codes were compared with respect to their prediction of interface shear resistance for clustered shear connectors. The database of 162 push-off tests was used to compare predicted interface shear resistance using four design codes versus the measured shear resistance as shown in Figure 7. The database includes test data obtained from 11 different data sources that cover variations of design parameters, such as shear pocket shape, interface shear areas, concrete compressive strength, reinforcement ratio, clamping stress, connector type, and yield strength. The values/ranges of these parameters were:

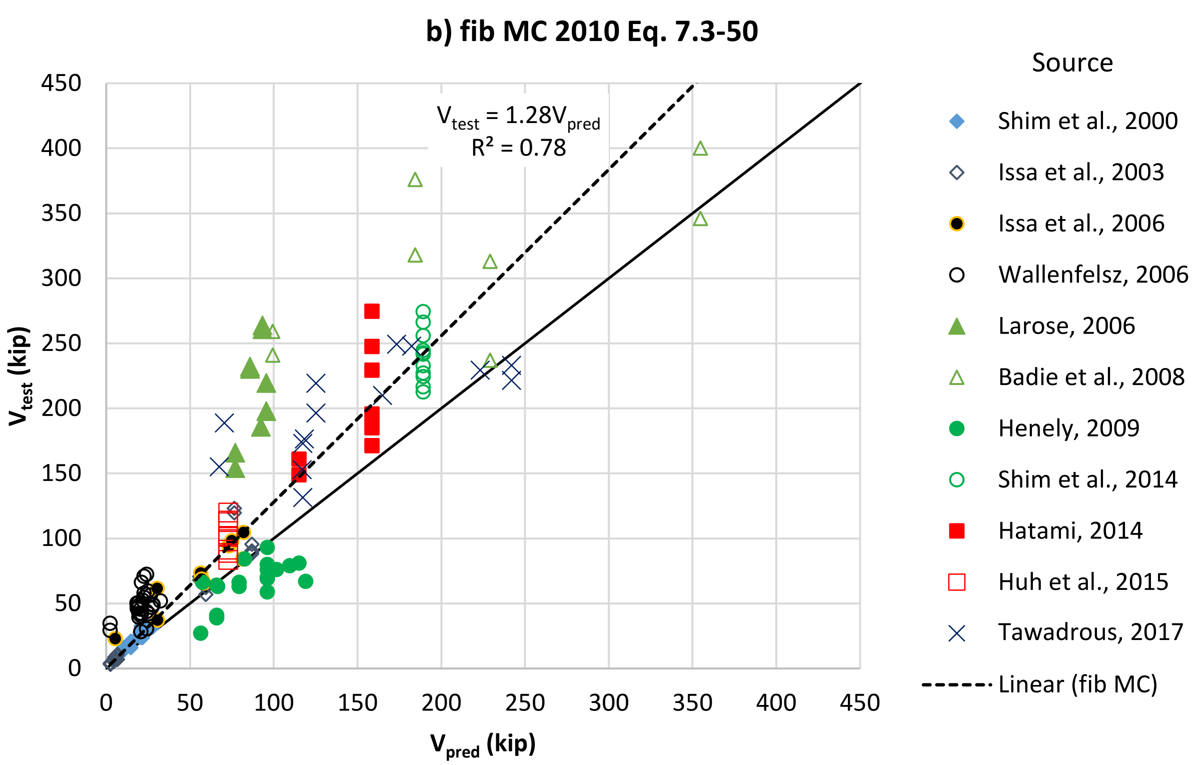
1. Pocket shapes include rectangular, circular, and beveled.
2. Interface shear area ranges from 4 to 178.25 in.^{2};
3. Average concrete compressive strength ranges from 2.5 to 11 ksi;
4. Reinforcement ratio (ρ) ranges from 0 to 11%;
5. Clamping stress ($\rho.f_y$) ranges from 0 to 5.9 ksi;
6. Shear connectors include reinforcing steel bars, studs, and threaded rods;
7. Yield strength of shear connectors ranges from 49 to 130 ksi.

The predicted shear resistance values in Figure 7 were calculated assuming the following:

- Strength reduction factor of 1.0;
- Measured not specified values of concrete compressive strength;
- Corresponding code provisions are used for calculating different parameters, such as material limits, and shear friction coefficient;
- Location of critical section is at shear pocket-haunch interface (see Figure 3)

For more detailed information about the dataset, refer to Tawadrous (2017).


a) AASHTO LRFD (2014) Eq. 5.8.4.1
$V_{test} = 1.27V_{pred}$
$R^2 = 0.65$
V_{test} (kip)
V_{pred} (kip)
0
50
100
150
200
250
300
350
400
450
Source
Shim et al., 2000
Issa et al., 2003
Issa et al., 2006
Wallenfelsz, 2006
Larose, 2006
Badie et al., 2008
Henely, 2009
Shim et al., 2014
Hatami, 2014
Huh et al., 2015
Tawadrous, 2017
Linear (AASHTO LRFD)


b) fib MC 2010 Eq. 7.3-50
$V_{test} = 1.28V_{pred}$
$R^2 = 0.78$
V_{test} (kip)
V_{pred} (kip)
0
50
100
150
200
250
300
350
400
450
Source
Shim et al., 2000
Issa et al., 2003
Issa et al., 2006
Wallenfelsz, 2006
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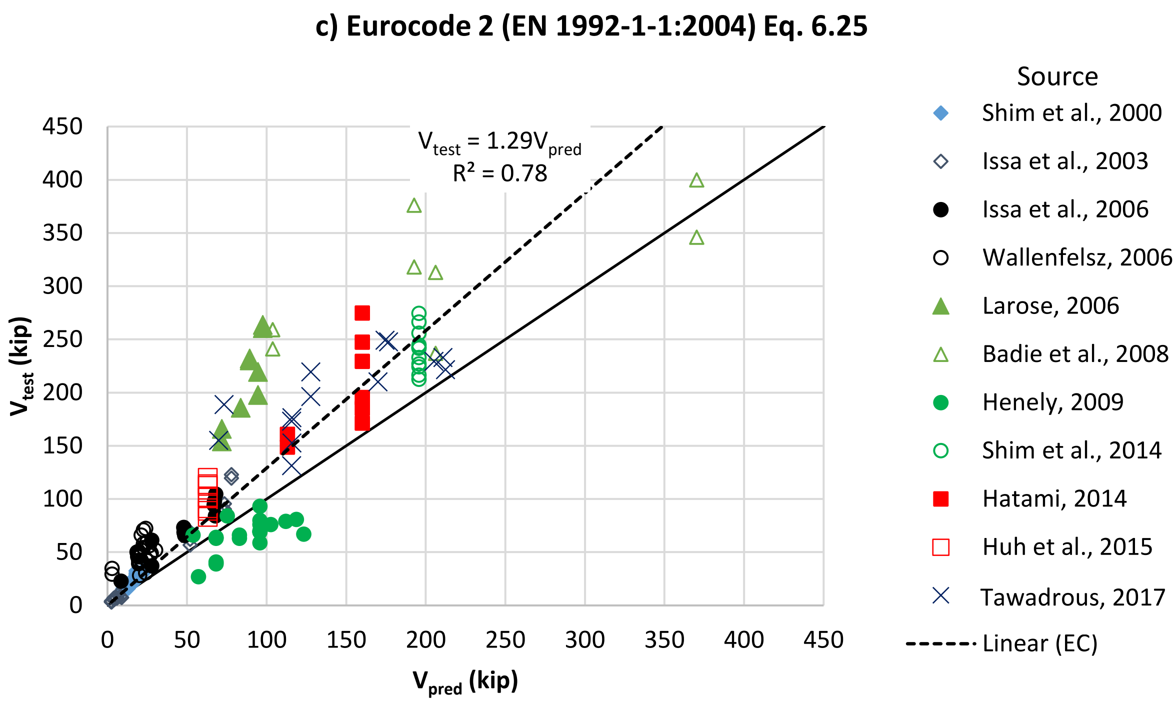
Badie et al., 2008
Henely, 2009
Shim et al., 2014
Hatami, 2014
Huh et al., 2015
Tawadrous, 2017
Linear (fib MC)



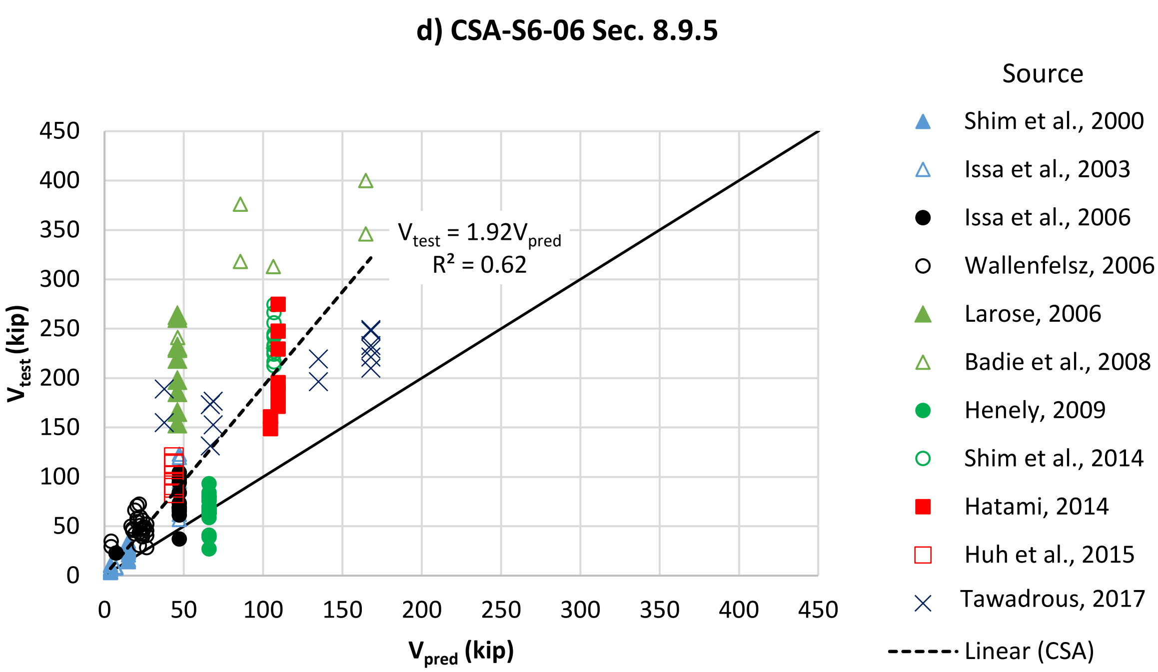


Figure 7: Measured versus predicted interface shear resistance using different design codes

Table 6 lists the mean, STD, and COV values of V_{test}/V_{pred} as well as the percentage of data points with $V_{test}/V_{pred} \geq 1.0$ for each design code to evaluate their relative accuracy.

Table 6: Summary of the accuracy and consistency of different code predictions

	V_{test}/V_{pred}			
	***fib* MC 2010**	**AASHTO LRFD 2014**	**EC2-2004**	**CSA-S6-2006**
Mean	1.49	1.45	1.54	2.07
STD	0.58	0.76	0.56	1.05
COV	0.39	0.52	0.36	0.50
UEV (%)	86	70	88	93

STD = standard deviation; COV = coefficient of variation; UEV (%) = percentage of underestimated values ($V_{test}/V_{pred} \geq 1.0$).

Discussion

Comparing the plots presented in Figure 7 a) to d) indicated that all code provisions conservatively underestimate the interface shear resistance of clustered shear connections. Although AASHTO LRFD 2014, EC-2, and CSA-S6 are based on the same shear friction model that was first developed by Birkeland and Birkeland (1966), these three codes provided different predictions as the slope of the trend-line was 1.27, 1.29, and 1.92, respectively. This is mainly due to differences in material strength limits specified in each of the code provisions. AASHTO LRFD, *fib* MC 2010, and EC-2 provided close prediction results with a difference (in terms of the mean) of less than 6.5% between all three of them. AASHTO provided the closest results to the measured interface shear resistance value with trend-line slope of 1.27. However, the European code and *fib* MC provided the most consistent results when compared to other codes as they have the least standard deviation (STD) and coefficient of variation (COV), and the highest R^2 values.

On the other hand, the Canadian code (CSA-S6) provided the most conservative predictions to interface shear resistance where the predicted interface shear resistance values were almost double the measured values. This may be attributed to ignoring the concrete contribution for monolithically cast concrete with compressive strength greater than 4 ksi. However, other codes such as EC-2 and *fib* MC use upper interface shear resistance limit that is a function of the concrete compressive strength (f_c), instead of an absolute limit. When the percentage of data points of the 162 push-off tests with $V_{test}/V_{pred} \geq 1.0$ was checked, it was found that the Canadian code (CSA-S6) provided the highest percentage (93%) among the other codes, which means that 93% of the predicted values were lower than the test values. The EC-2 and *fib* MC 2010 provided close percentages of 88 and 86%, respectively. AASHTO LRFD provided the smallest percentage where only 70% of the predicted values were lower than the test values.

Conclusions

Based on the analytical evaluation presented in this paper on the interface shear resistance of clustered shear connectors, the following conclusions can be made:

- Concrete compressive strength (f_c) has no significant effect on interface shear resistance when reinforcement ratio across the interface is less than 4%. However, concrete compressive strength has slight correlation with interface shear resistance ($R^2 = 0.22$) when reinforcement ratio exceeds 4%;
- Interface shear resistance of clustered shear connectors increases as the reinforcement ratio across the interface increases, however, the tensile yield strength of shear connectors does not have a significant effect on interface shear resistance.

- AASHTO LRFD, *fib* MC 2010, and EC-2 provisions provide close predictions for interface shear resistance of clustered shear connectors (mean V_{test}/V_{pred} of 1.45, 1.49, and 1.54, respectively) while, CSA-S6 provides the most conservative predictions (mean of V_{test}/V_{pred} = 2.07);
- EC-2 and *fib* MC code provide the most consistent predictions as they resulted in the least COV value for V_{test}/V_{pred} are 36 and 39 %, respectively. On the other hand, AASHTO LRFD and CSA-S6 provide the most scattered predictions as their COV for V_{test}/V_{pred} are 52% and 50%, respectively.
- Interface shear provisions of EC-2 and *fib* MC 2010 codes are recommended for predicting interface shear resistance of clustered shear connectors, as more than 85% of the database was well predicted by these two codes.

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