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STATIC STRENGTH OF HEADED SHEAR STUD CONNECTORS USING FINITE ELEMENT ANALYSIS

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Abstract: The use of composite structures in highway bridges has become a widespread practice and for developing composite action between steel beam and concrete slab, headed shear studs are widely used. The strength and ductility of these shear connectors greatly influence the capacity of composite structures. The objective of this paper is to investigate the static strength of headed shear stud connectors. For this purpose, a three-dimensional finite element model of push out test was developed using general purpose finite element software ABAQUS. The FE model included both geometric and material nonlinearities. The developed FE model was able to predict shear capacity and load-slip behavior of the shear stud reasonably accurately. After validation against test results, an extensive parametric study has been performed to investigate different parameters such as variations of concrete strength and stud diameter. Results from the FE analysis were also compared with the current code of practices, such as EC4 and CSA S6-14. The parametric study showed that CSA S6-14 usually overestimates the static strength of headed shear stud connectors.

1. INTRODUCTION

To transmit the developed shear forces across the steel-concrete interface, shear connectors are commonly used in steel-concrete composite bridges. These shear connectors are welded on top of steel beam and primary purpose is to prevent horizontal movement and separation between steel beam and concrete slab which allows them to act as one unit. The most common headed shear studs used in composite bridges are 19 and 22 mm. Many shear studs are required in the area of high shear zone to provide full shear connection resulting a long welding time and difficulty to remove a deteriorated slab and also a dense distribution of shear studs can cause difficulty for the workers in case of smaller shear studs are used (Lee et al. 2005). As a result, use of larger shear studs, such as 25, 27 and 30 mm are now getting attraction from engineers , however a very few works have been done on larger shear studs. The main factors affecting the behavior of shear connections are strength of concrete and connector and experimental push-out tests are done to evaluate both shear capacity and load-slip behavior of shear connectors. Since full-scaled push-out tests are very expensive and time consuming, analytical procedures are the best alternative once it has been verified against test results. In this paper, a three-dimensional finite element model of push out test was developed using general purpose finite element software ABAQUS. This FE model is used to predict shear capacity and load-slip behavior of both small (19 and 22 mm) and larger headed shear studs (25, 27 and 30 mm) used in composite bridges. The effect of concrete strength on static shear strength of stud is also evaluated and compared with current bridge design codes such as CSA S6-14 and EC 4.

2. BACKGROUND

2.1 Brief Review of Previous Works

To find out static strength and load-slip behavior of headed shear stud connectors, push-out tests are mostly used worldwide. A typical push-out specimen consists of a steel beam on which shear connectors are welded on both flanges and embedded in concrete slab. The specimen is loaded until failure and the recorded ultimate load is divided by the number of shear studs to get static strength of shear stud connectors. It is assumed that the load is transmitted from steel beam to concrete slab uniformly through shear stud connectors for simplicity (Viest 1956). Slutter and Fisher (1966) tested 35 push-out specimens having the concrete slab connected with steel beam by 19 mm shear stud, 9 push-out specimens of 22.2 mm shear stud and 12 push-out specimens for channel shear connectors. Mainstone and Menzies (1967) conducted push-out tests on 11 specimens using 19 mm dia shear studs of 100 mm height. Badie et al. (2002) reported application of 31.8 mm diameter shear stud. It was found that the static strength of 31.8 mm shear stud was almost double than that of 22 mm shear stud. Thus, fewer studs would be required in design. Tests on larger shear studs, such as 25, 27 and 30 mm were carried out by Lee et al. (2005) and the static shear strengths from the tests were compared with equations provided by EC 4 and AASHTO LRFD.

2.2 Static Design Specifications

The design provisions in American Association of State Highway and Transportation Officials (AASHTO LRFD) and Canadian Standards Association (CSA S6-14) are based on the research done by Ollgaard et al. (1971) for static shear strength. CSA S6-14 states that the factored shear resistance, q_r , of a headed stud shear connector with h/d ≥ 4 shall be taken lesser of two quantities in the following Equation 1.

[1] $$q_r = 0.5\, ø_{sc}\, A_{sc} \sqrt{f'_c Ec} \;\leq\; ø_{sc}\, F_u\, A_{sc}$$

where F_u = minimum tensile strength of the stud steel, A_{sc} = cross-sectional area of one stud shear connector and f'_c = concrete compressive strength. The left-hand side of the inequality of Equation 1 represents the shear stud strength as affected by modulus of elasticity and compressive strength of concrete while the right-hand side represents the stud strength, which is a function of tensile strength of the shear stud (Jayas and Hussain 1988). According to Eurocode-4, the static strength of shear stud in composite beam should be taken as the lesser of Equation 2 and Equation 3.

[2] $$q_r = 0.8fu\ (\ d^2/4)/\gamma_v$$

[3] $$q_r = 0.29\alpha d^2\sqrt{(f_{ck}Ecm)}\ /\gamma_v$$

where fu = ultimate strength of steel, f_{ck} = cylindrical compressive strength of concrete, Ecm = Elastic modulus of concrete, γ_v is a partial safety factor (= 1.25) and $\alpha = 0.2\left(\frac{h}{d}+1\right) \leq 1.0$; h and d are the overall height and diameter of the stud respectively.

3. FINITE ELEMENT MODELING

To obtain accurate results from a successful numerical model, it is very important to model all the components of push-out specimen such as concrete slab, steel beam, rebar and shear studs. A general purpose finite element modeling package, ABAQUS, has been selected for this purpose and both geometric and material nonlinearity are included in the developed FE model.

3.1 FE Model Geometry

The push-out specimen used in the test of Lee et al. (2005), which is in accordance with standard push-out specimen in Eurocode-4, has been chosen for the development of FE modeling as shown in Figure 1. The push-out specimen consists of concrete slab, steel beam, rebar and headed shear studs. The thickness of steel beam and concrete slab are 14 mm and 200 mm respectively. Two smaller studs i.e. 19 mm and 22 mm and three larger headed shear studs i.e. 25 mm, 27 mm and 30 mm have been used in this paper. Due to the symmetry of push-out specimen, a quarter of the whole model has been used and appropriate boundary conditions are applied to replicate the whole model.

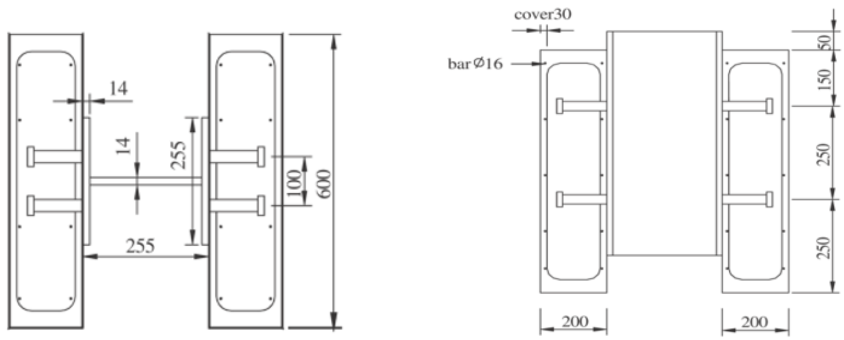

Figure 1: Push-out Model Geometry (Lee et al. 2005)

The selected dimensions for headed shear studs used in FE analysis are shown in the following Table 1.

Table 1: Dimensions of Headed Shear Stud used in FE Analysis

Diameter of Stud (mm)	Stud Head Height (mm)	Stud Overall Height (mm)	Stud Head Diameter (mm)
19	9	125	31
22	11	155	35
25	11	155	38
27	12	155	41
30	12	155	44

The shear studs are modeled using the exact geometry as shown in Table 1 to consider the complicated contact interactions and fracture mechanisms. Reinforcement bars are modeled as solid parts and embedded in concrete slab; all the nodes are tied to concrete slab allowing no slip between them.

3.2 Analysis Procedure and Load Application

ABAQUS/Explicit is adopted in this research since it is suitable for impact problems, progressive damage, failure of material, large deformation, contact interaction. The analysis time of this analysis method can be reduced by using mass scaling or increasing loading rate. Both geometric and material nonlinearities are introduced in the FE analysis. All the nodes lying on the load surface were constrained to reference point and displacement controlled loading was applied at that reference point till failure. To do so, MPC constraint has been used between load surfaces (top surface of steel beam) and reference point. The loading rate has been decided appropriate on the basis of quasi-static assumption in which the load is applied so slowly that the structure also deforms very slowly as to appear a static condition. In Abaqus, an analysis can be called quasi-static analysis if the ratio of internal energy and kinetic energy is at least 5% or greater. This was checked during analysis.

3.3 Contact and Interaction

Surface-to-surface contact (Explicit) procedure was used in Abaqus/Explicit with normal behavior ("Hard" conatct) and tangential behavior ("penalty" formulation). A frictionless interaction has been used between steel beam and concrete slab. The reason for using frictionless interaction is to ensure the proper test condition beacause in tests of Lee et al. (2005), the bonding at the interface between the flanges of the steel beam and the concrete slab was prevented by greasing the flange. The mechanism assumed in this interaction is that the load will be transferred from steel beam to the headed shear studs and, eventually to the concrete slab. In case of concrete slab-shear stud interface, shear studs have been selected as master surface and concrete slab as slave surface since shear studs are more stiffer.

3.4 Boundary Conditions

Boundary conditions are very important for the simulation of experimental program and any inappropriate boundary conditions may cause completely different and wrong results. The boundary conditions are selected on the basis of accurate load-slip behavior prediction of headed shear stud. All the nodes lying in surface-1 are restricted from moving in X direction and rotation about Y and Z axis are also restrained while all the nodes in the middle of the steel beam web, designated as surface 2, are restricted from moving in Z direction, as well as rotation about X and Y axis are restrained also. At the bottom surface of concrete slab denoted by surface 3, all translational and rotational movements are restrained as shown in Figure 2.

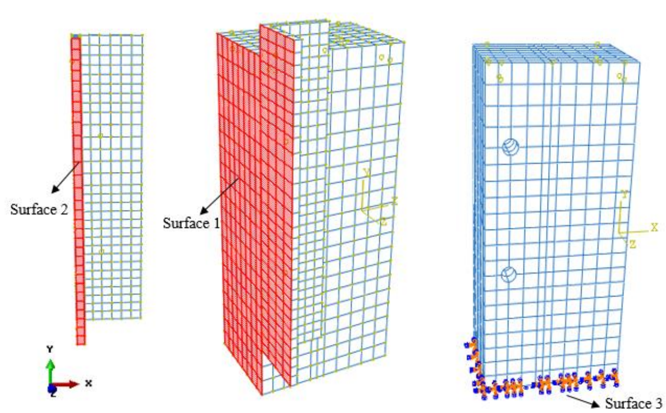

Figure 2: Boundary Conditions for FE Model

3.5 Material Properties

As mentioend earlier, two types of headed shear studs are used in order to investigate load-slip characteristics and static strength. Shear studs of 19 mm and 22 mm diameter are most common in steel-concrete composite bridges. 25 mm, 27 and 30 mm shear studs known as larger shear studs have also been investigated because currently there ae no guidelines in CSA S6-14 for the shear studs greater than 25 mm diameter. The yield and ultimate strength of headed shear studs for 19 and 22 mm diameter are chosen as 350 and 480 MPa as used in test of Gattesco and Giuriani (1996). For 25, 27 and 30 mm dia shear studs, yield and ultimate strength are chosen from the test of Lee et al. (2005). The nonlinear behavior of the concrete material as shown in Figure 3 was used by Nguyen and Kim (2009) which represents uniaxial stress-strain of concrete. In this paper, this uniaxial stress-strain curve of concrete has been used with slight modifications. There are three parts in this stress-strain curve; in first part, stress increases linearly up to 0.4 f'c. The young's modulus is calculated based on the following Equation 4 as mentioned in CSA A23.3-14.

[4] $E_{concrete} = 4500\sqrt{f'c}$

where $f'c$ and $E_{concrete}$ are in MPa. The second part of the curve is an ascending part up to $0.9f'c$, where $f'c$ is the 28-days concrete cylindical compressive strength. The peak stress is used as $0.9f'c$, as suggested in CSA A23.3-14. The strain ε1 related to $0.9f'c$ has been taken as 0.0022 and Poisson's ratio as 0.2 for concrete. The third part of the curve is a descending part up to r f'c where the value of r is the reduction factor and the value of r is taken from the study of Ellobody et al. (2006) and the ultimate strain of concrete is used as 0.0035, as suggested by CSA A23.3-14. For concrete in tension, the tesile stress is assumed to increase linearly till crack and f_t is calculated based on Equation 5 given in CSA A23.3-14:

[5] $f_t = 0.6\sqrt{f'c}$

where f_t and $f'c$ are in MPa. Finally, f_t, tensile stress decreases linearly to zero. The strain at zero tensile stress has been taken as 0.005 as used by Nguyen and Kim (2009). For both structural and reinforcement steel, bi-linear stress-strain relationships have been assumed representing a simple elastic-plastic model. Poisson's ratio is taken as 0.3 for structural and reinforcement steel material. The material properites used in the tests of Lee et al. (2005), for structural and reinforcement steel, are used in the FE analysis.

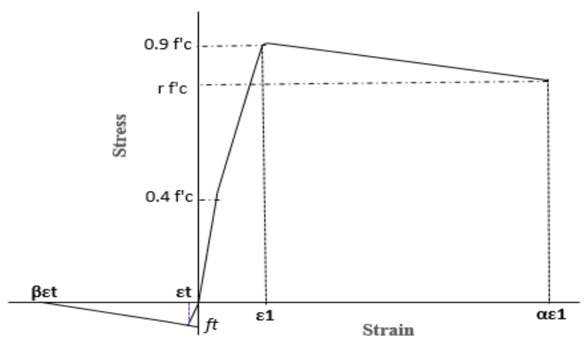

Figure 3: Stress-strain Relationships for Concrete Material

3.6 FE Mesh

In order to achieve an accurate results, three dimensional solid elements, particularly hexahedrals, are used in this study. Solid elements can be used for linear and nonlinear simulations involving contacts, plasticty successfully. As shown in Figure 4, for concrete slab, steel beam and headed shear studs, a three-dimensional eight-node element (C3D8R) is selected. C3D8R is an eight node brick element with reduced integration and each node has three translational degrees of freedom. This element type also prevents mesh locking when material response is incompressible by providing a constant volumetric strain and it is very suitable in case of nonlinearity problems. T3D2 element with linear approximation of displacement was used for rebars and this truss element type has two nodes and three translational degrees of freedom. The mesh size near stud was reduced to get more accurate results since that area is our interst to see the effects of applied displacement. It can be noted here that relative displacement was measured between the nodes on steel beam and concrete slab near the stud. This is another reason to choose finer mesh near the stud.

Figure 4: Mesh of Concrete Slab, Shear Stud and Steel Beam

3.7 FE Model Validation

Figure 5 presents the finite element model validation of the push-out test. Load versus relative displacement curve from the test results of Gattesco and Giuriani (1996) is compared with results from FE analysis. The shear stud diameter for the test was 19 mm. In their tests, compressive cube strength of concrete (fcu) was used as 32.5 MPa and compressive cylinder strength of concrete is assumed as 26 MPa (0.8 fcu). The ultimate slip value was reported 9.7 mm in test of Gattesco and Giuriani (1996) while from FE analysis, it is found as 9.61 mm. The slip at which the load has reduced by 10% from its peak is used as ultimate slip in the developed FE model.

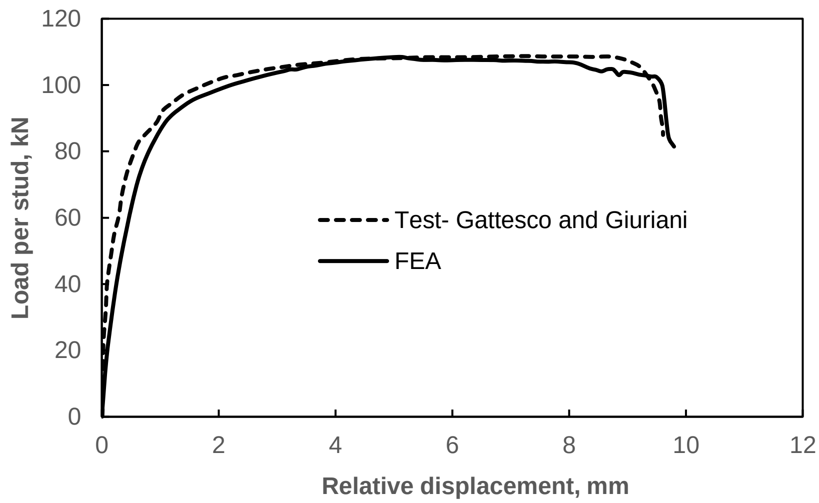

Figure 5: Validation of FE Model with Test Results (19 mm Shear Stud)

Lee et al. (2005) performed nine push-out tests on three stud diameters of 25, 27 and 30 mm to investigate experimentally static and fatigue behavior of large shear stud connectors. For each diameter of shear stud, three tests were conducted, as shown in the Table 2. Finite element analyses of these nine specimens were conducted. Table 2 compares the test results with the FE analysis results. A good aggrement of both static strength and ultimate slip are found. It is important to note here that since three tests were performed for each diameter, the average value is used for comparison purpose.

Table 2: Comparison of FE Analysis Results with Test Results of Lee et al. (2005)

Diameter of Stud (mm)	Test Results					FE Analysis Results	
	Specimen	Staic strength (kN)	Average	Ultimate slip (mm)	Average	Staic strength (kN)	Ultimate slip (mm)
25	ST25B1	176.4		6.33			
	ST25B2	176.7	180.13	6.72	6.79	175.394	8.59
	ST25B3	187.3		7.31			
27	ST27C1	208.2		9.19			
	ST27C2	238.5	211.2	8.36	8.82	208	9.12
	ST27C3	186.9		8.92			
30	ST30C1	222.8		9.39			
	ST30C2	240.0	232.27	9.24	9.36	242.92	10.02
	ST30C3	234.0		9.46			

4. PARAMETRIC STUDY

A parametric study is conducted to study the effects of concrete strength and shear stud diameter on the shear capacity of stud connectors. Five different concrete strengths such as 25, 30, 35, 40 and 45 MPa are selected. Also, five different shear stud diameters, 19, 22, 25, 27 and 30 mm, were selected for the parametric study. Comparison of FE analysis results with both Canadian (CSA S6-14) and European code (EC-4) are shown in Figure 6. As stated earlier, both Canadian and American have same provisions for shear strength of stud connectors. It is observed from Figure 6 that European code generally provides better prediction of shear capacity of headed shear stud.

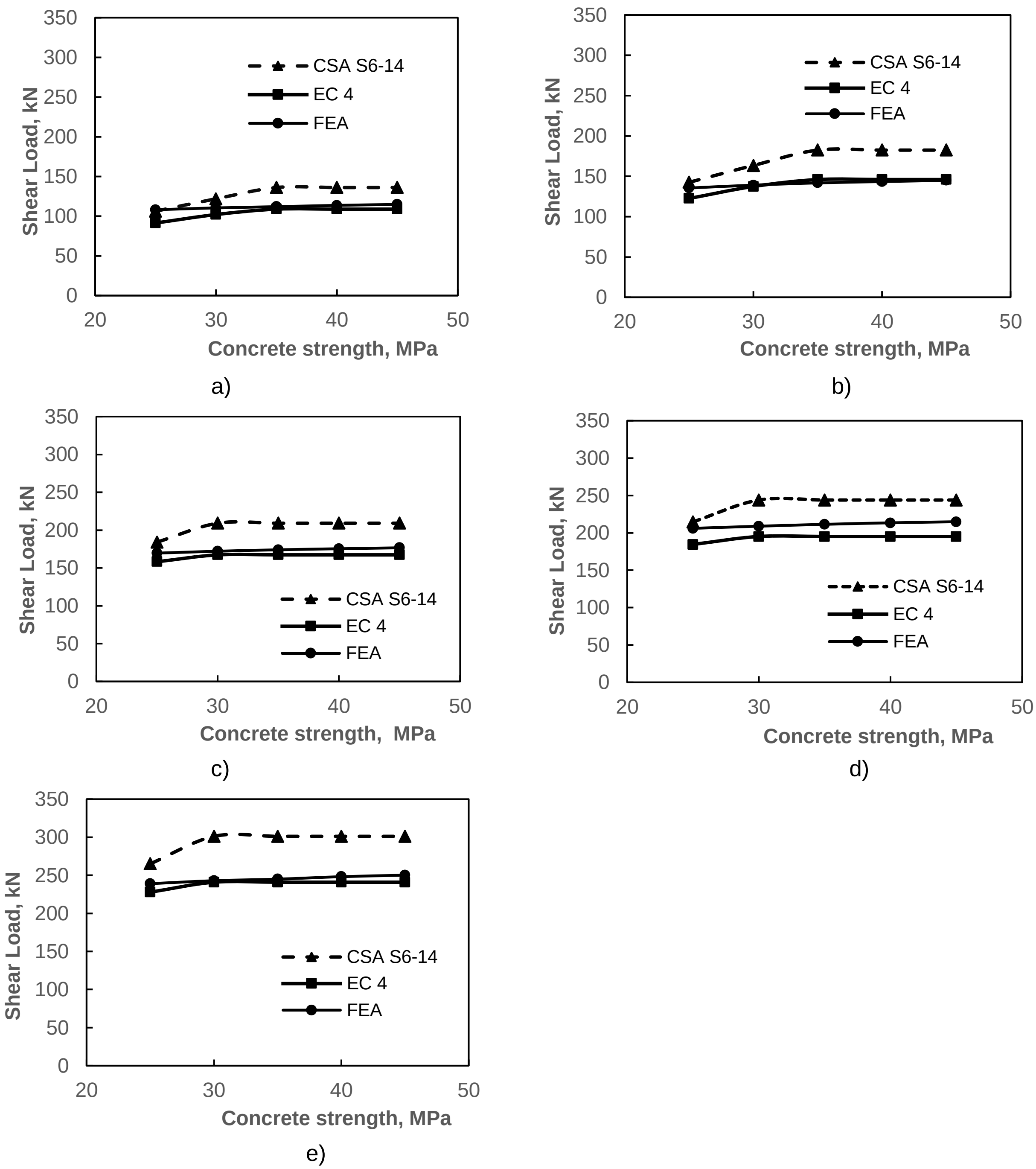

Figure 6: Comaprison of Shear Capacity Obtained from FE Analysis with CSA S6-14 and EC-4 for a) 19 mm, b) 22 mm, c) 25 mm, d) 27 mm and e) 30 mm Headed Shear Stud

Table 3-7 compare the shear capacities of different shear stud connectors obtained from FE analysis with the predicted values from different codes.

Table 3: Variation of Shear Capacity for 19 mm Headed Shear Stud

Concrete Strength (MPa)	CSA S6-14	EC 4	FEA	$P_{FEA}/P_{CSA\ S6\text{-}14}$	%	$P_{FEA}/P_{EC\ 4}$	%
25	106.32	91.37	108.09	1.0166	1.6609	1.1829	18.2935
30	121.90	102.07	110.44	0.9059	9.4036	1.0819	8.1996
35	136.09	108.88	111.98	0.8228	17.7172	1.0285	2.8534
40	136.09	108.88	113.63	0.8350	16.5037	1.0437	4.3703
45	136.09	108.88	114.84	0.8438	15.6154	1.0548	5.4808

Table 4: Variation of Shear Capacity for 22 mm Headed Shear Stud

Concrete Strength (MPa)	CSA S6-14	EC 4	FEA	$P_{FEA}/P_{CSA\ S6\text{-}14}$	%	$P_{FEA}/P_{EC\ 4}$	%
25	142.55	122.51	135.49	0.9505	4.9482	1.1060	10.6030
30	163.44	137.39	139.08	0.8509	14.9021	1.0123	1.2301
35	182.46	145.97	141.84	0.7774	22.2628	0.9717	2.8285
40	182.46	145.97	143.43	0.7861	21.3922	0.9826	1.7403
45	182.46	145.97	145.00	0.7947	20.5277	0.9934	0.6596

Table 5: Variation of Shear Capacity for 25 mm Headed Shear Stud

Concrete Strength (MPa)	CSA S6-14	EC 4	FEA	$P_{FEA}/P_{CSA\ S6\text{-}14}$	%	$P_{FEA}/P_{EC\ 4}$	%
25	184.08	158.19	169.61	0.9214	7.8609	1.0721	7.2138
30	209.11	167.29	172.09	0.8230	17.7018	1.0287	2.8727
35	209.11	167.29	173.99	0.8320	16.7954	1.0401	4.0058
40	209.11	167.29	175.39	0.8388	16.1247	1.0484	4.8442
45	209.11	167.29	176.63	0.8447	15.5319	1.0559	5.5851

Table 6: Variation of Shear Capacity for 27 mm Headed Shear Stud

Concrete Strength (MPa)	CSA S6-14	EC 4	FEA	$P_{FEA}/P_{CSA\ S6\text{-}14}$	%	$P_{FEA}/P_{EC\ 4}$	%
25	214.71	184.52	205.98	0.9593	4.0670	1.1163	11.6284
30	243.91	195.13	208.89	0.8564	14.3574	1.0705	7.0532
35	243.91	195.13	211.4	0.8667	13.3284	1.0834	8.3395
40	243.91	195.13	213.4	0.8749	12.5084	1.0936	9.3645
45	243.91	195.13	214.88	0.8810	11.9016	1.1012	10.1230

Table 7: Variation of Shear Capacity for 30 mm Headed Shear Stud

Concrete Strength (MPa)	CSA S6-14	EC 4	FEA	$P_{FEA}/P_{CSA\ S6\text{-}14}$	%	$P_{FEA}/P_{EC\ 4}$	%
25	265.07	227.80	238.98	0.9016	9.8420	1.0491	4.9085
30	301.12	240.90	242.92	0.8067	19.3285	1.0084	0.8394
35	301.12	240.90	244.95	0.8134	18.6552	1.0168	1.6810
40	301.12	240.90	248.20	0.8243	17.5747	1.0303	3.0316
45	301.12	240.90	250.09	0.8305	16.9479	1.0382	3.8151

From all the tables, it is observed that Canadian standard, CSA S6-14 generally overestimates the shear capacity of headed shear studs. The overestimation increases with the increase of concrete strength. Thus, for 19 mm shear stud diameter, as observed in Table 3, Canadian standard, S6-14 overestimates the shear strength as much as 17.7% when 35 MPa concrete is used. In addition, EC 4 ususally underestimates the shear capacities of the shear studs. For 19 mm dia shear stud, the underestimation is up to 18.3%, but for larger shear studs Equations 2 and 3 proposed in EC 4 are found to provide very close estimations when compared to the strengths obtaied from FE analysis.

The shear load per stud versus slip relationship of the push-out specimen for 22 mm shear stud is shown in the following Figure 7. From this figure, it is clear that shear capacity increases with the increase of concrete strength but ultimate slip decreases.

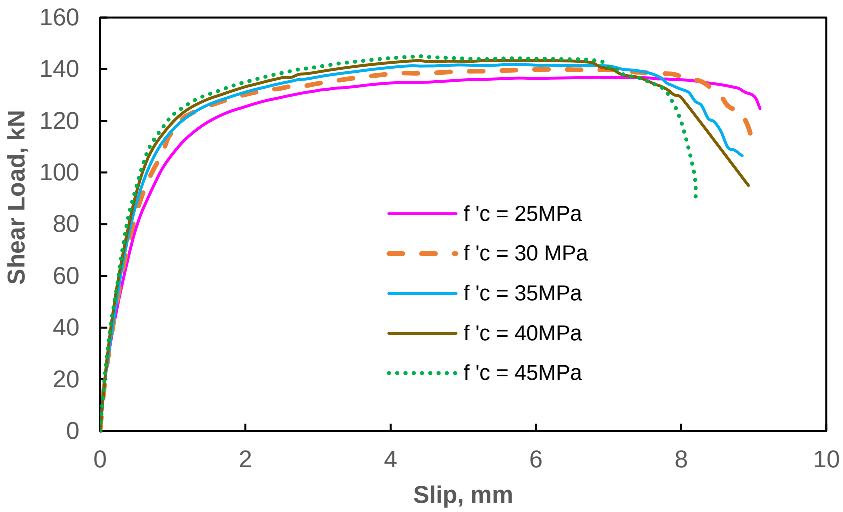

Figure 7: Effect of Concrete Strength on Load-slip Behavior for 22 mm Shear Stud

In the test of Lee et al. (2005), shank failure mode was reported for all stud diameters, such as 25, 27 and 30 mm. Similar failure is also found from the FE analysis, as shown in Figure 8. It can be noted here that for smaller shear stud, such as 19 mm, same type of failure mode was observed in the test of Gattesco and Giuriani (1996).

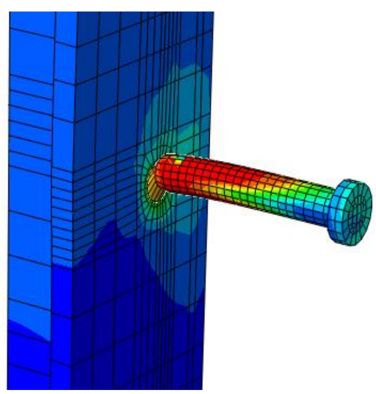

Figure 8: Shank Failure Mode

5. CONCLUSION

An extensive parametric study with different stud diameters and concrete strength was performed using the develoed FE model to investigate the shear capacity and load-slip behavior of both small and larger shear studs to shed some light on this issue. The shear capacity and failure modes are well matched with experiments and all the failure modes were accurately predicted by FE analysis. It has been found that shear capacity increases with the increase of concrete strength but the slip decreases. With the increase of stud diameter, the ultimate slip increases for a certain concrete strength. From the parametric study, Canadian Standard, CSA S6-14 is found to overestimate the static strength of headed shear stud up to 22.3% while the European code, EC4 ususally gives conservative estimation of shear capacity of headed shear stud.

Acknowledgements

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